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Evaluation of Seismic Actions on a Building Founded on Piles through Dynamic Analysis of Soil-Structure Interaction

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ABSTRACT

A tall r.c. building in the city of Napoli has recently undergone a seismic vulnerability assessment, in order to satisfy its public function fulfilling the requirements of the new Italian Building Code. The study is focused on the inertial response of the building, accounting for soil deformability and damping. To this aim, the overall structure is idealized as a simple oscillator supported on frequency-dependent springs and dashpots.

Introduction

Literature studies (e.g. Han, 2002) have shown that the seismic response of a tall building supported on a pile foundation may be difficult to be predicted correctly, if the complex dynamic interaction problem is not handled with care. In such a case, the assessment of the seismic vulnerability of the building would be inaccurate, hence expensive and likely useless retrofitting could be undertaken to meet the seismic safety requirements. An adequate procedure to consider the soil-foundation-building interaction is based on the sub-structures method (Kausel *et al*. 1978; Gazetas, 1984). In this work, such a procedure was to a tall building supported by piles and located in the eastern area of Naples (Italy), for which a seismic vulnerability assessment conventionally performed by applying the standard code spectra to a fixed-base structural model, had evidenced a lack of capacity of the seismic-resistant system (Bilotta *et al*., 2013a). Seismic action were then evaluated 'unconventionally', by properly modelling SSR effects and foundation compliance. The inertial response of the building was analysed following by a closed form solution based on the replacement oscillator method.

Case Study

The building under consideration (Figure 1a) is a 29-storey reinforced concrete tower, with a height of 107 m, built in the early '80s. The tower, with a stiffening core, is rigidly connected to a piled foundation by a r.c. box structure, consisting of a lower raft as thick as 1 m and an upper 40 cm slab, joined by vertical reinforced concrete walls 6 m high. The 82 piles are unevenly distributed on a large area of 3300 m^2 (Figure 2); they were drilled in alluvial and volcanic soils with a length of 42 m and a diameter ranging between 1800 mm and 2200 mm (Mancuso et al., 1999).

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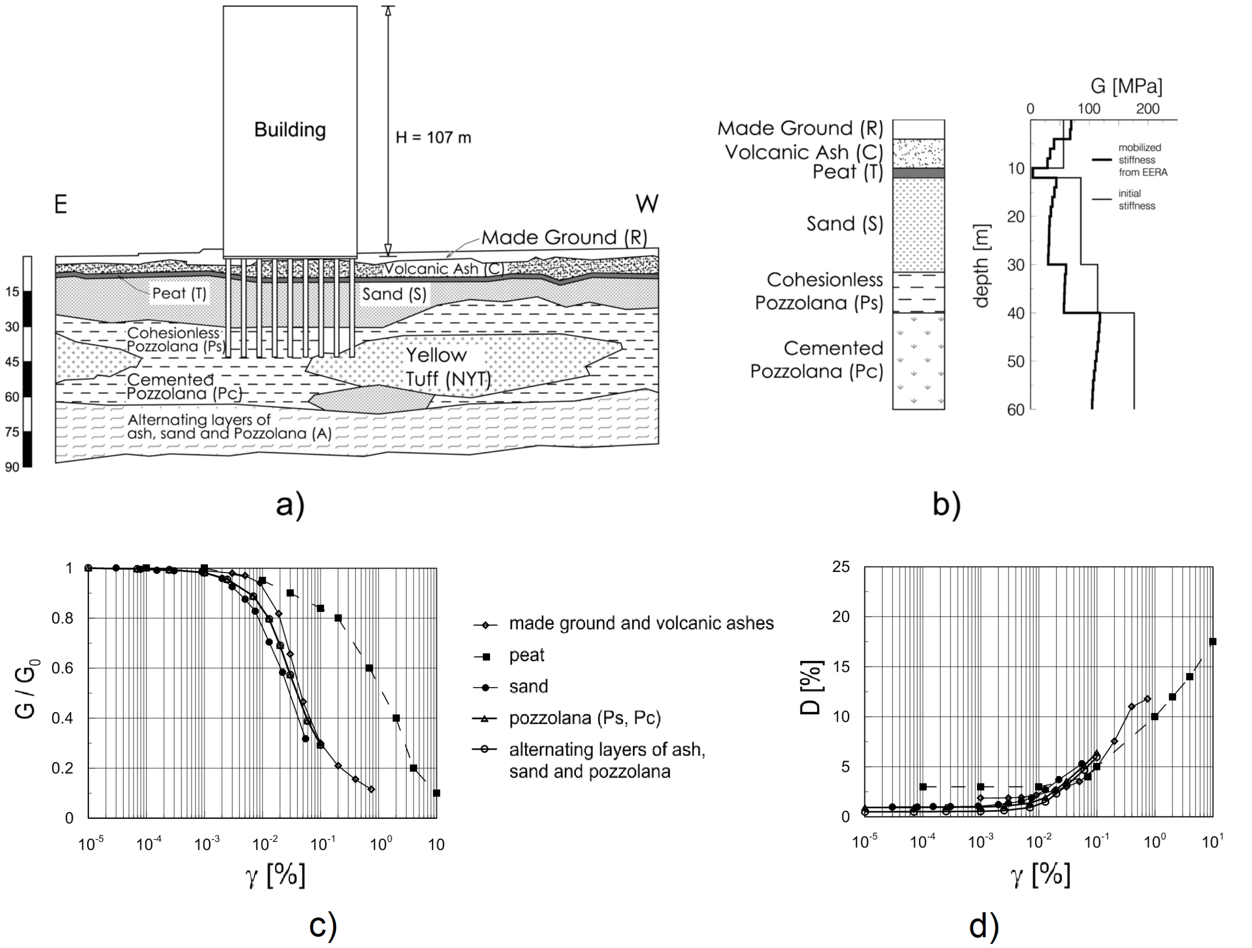

Figure 1. Sketch of the building and ground conditions (a); material properties (b, c, d).

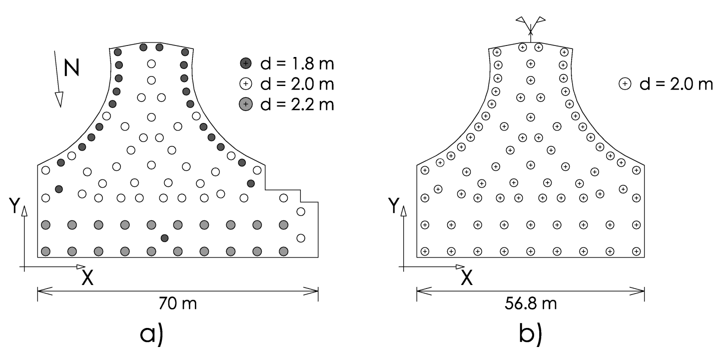

Figure 2. Layout of foundation piles: a) actual foundation; b) simplified model

On the basis of available geotechnical investigations (Vinale, 1988), a regular layering was adopted (Figure 1b) to carry out one-dimensional seismic response analyses with the equivalent linear approach in the frequency domain, by using the code EERA (Bardet et al., 2000), as described in Bilotta *et al*. (2015). The adopted curves for normalized stiffness decay and damping ratio are shown in Figures 1c and 1d. A visco-elastic bedrock was assumed at 60 m depth, with shear wave velocity $V_{S,b}$ = 800 m/s and damping ratio D = 0.5%.

Seven natural accelerograms were extracted from the European Strong Motion Database (ESD), compatible on the average with the code-specified spectrum for the limit state of life safety, through the software REXEL 3.5 (Iervolino et al. 2009). The selected input signals were all scaled to $a_{\max} = 0.19g$. Figure 3 shows the corresponding mean response spectrum (dotted line).

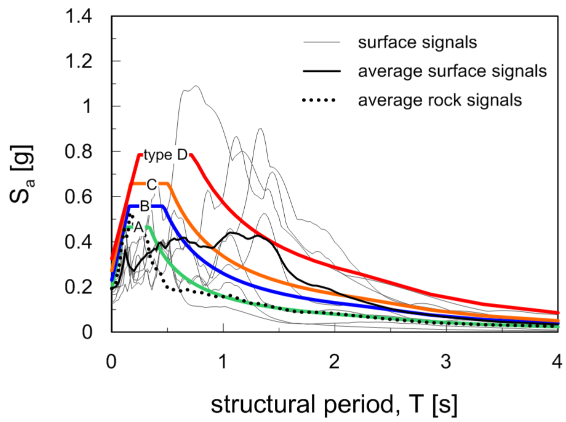

Figure 3. Mean input spectrum, output spectra from SRA, Italian code spectra.

In the same figure, the output spectra calculated at surface for each input signal are shown with thin grey lines and their average (black solid line) can be compared to the standard spectra specified by the Italian code (NTC, 2008) for different ground types (coloured lines). It may be observed that, for periods higher than 2 s, type C and D spectral ordinates overestimate the mean values predicted by the SSR analyses. Since the first period of the fixed base structure is 2.47 s, the beneficial effect of considering the site-specific seismic response analysis is apparent. On the average, the mobilized normalized stiffness G/G_0 never resulted less than about 60% (Figure 1b), showing that the shear strain level induced in the ground was lower enough than the shear-volumetric coupling threshold, that justifies using an equivalent linear approach in SSR analysis.

Dynamic Soil-structure Interaction

FE analyses of pile-soil kinematic interaction were preliminarily carried out on the basis of the profile of the stiffness mobilized in the seismic response analysis (Fig. 1b) to define the Foundation Input Motion. Specifically, the above analyses were carried out in the frequency domain by means of code Dynapile 2.0 (Ensoft, 1999), based on the consistent boundary matrix method (Blaney *et al.* 1976), in which group effects are incorporated through frequency-dependent pile-to-pile interaction factors for both swaying and rocking oscillation of the foundation. The ratio of pile acceleration over free-field acceleration is plotted against excitation frequency in Figure 4, for both pile group and the single pile. Group effects due to kinematic interaction are negligible, as expected. Moreover, the scattered field motion between piles and soil is not significant, at least in the range of dominant frequencies for the problem at hand (0÷1 Hz). In other words, the Foundation Input Motion can be considered approximately coincident with the free-field motion.

The dynamic response of a building founded on piles embedded in a deformable soil may be different from that of a similarly excited structure resting on a rigid ground. The factors responsible of such a different behavior are: (i) the flexibility of the pile-foundation-system; (ii) the energy dissipated by wave radiation and soil damping. The most recurrent approaches on this subject in the literature can be grouped in two classes, where either the soil-foundation system is idealized through the so-called macro-element approach (e.g. Figini *et al*., 2012) or the structure is replaced by a simple oscillator supported on frequency-dependent springs and dashpots (e.g. Veletsos & Meek, 1974).

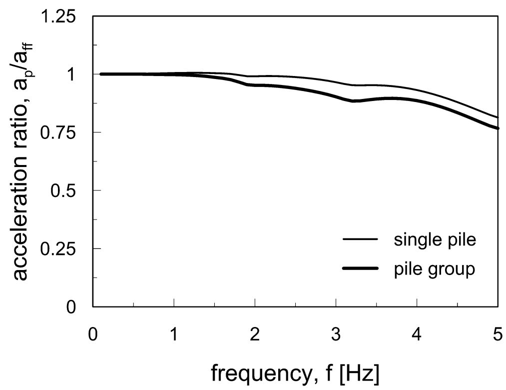

Figure 4. Results of kinematic interaction analyses

In this study, attention is focused on the latter approach, which is commonly referred to as the ‘replacement’ or the ‘equivalent’ oscillator. Figure 5 shows a simple oscillator on a flexible foundation; the dynamic compliance is modelled by two springs (K_X and K_θ) and a pair of dashpots (C_X and C_θ) associated to translational and rotational oscillations.

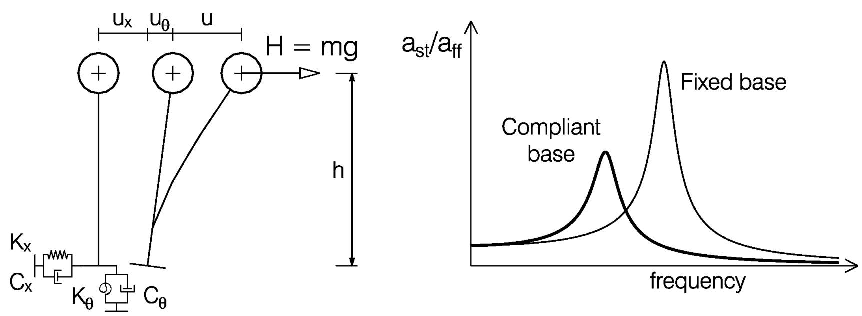

Figure 5. Replacement oscillator (left); transfer functions (right): fixed *vs* compliant base

A novel exact formulation for a structure resting on generic springs and dashpots has been recently proposed by Maravas et al. (2014). According to them, the damping and the natural frequency of the overall system for the swaying vibration mode along *x*-axis, for example, reads:

$$\tilde{\zeta}_X = S_X \left[\frac{\zeta_X}{\omega_X^2 \left(1+4\zeta_X^2\right)} + \frac{\zeta_\theta}{\omega_{\theta Y}^2 \left(1+4\zeta_{\theta Y}^2\right)} + \frac{\zeta_{CX}}{\omega_{CX}^2 \left(1+4\zeta_{CX}{}^2\right)} \right] \quad \text{(1a,b)}$$

$$\tilde{\omega}_X{}^2 = S_X / \left(1+4\tilde{\zeta}_X{}^2\right)$$

with:

$$S_X = \left[\frac{1}{\omega_X^2 \left(1+4\zeta_X^2\right)} + \frac{1}{\omega_{\theta Y}^2 \left(1+4\zeta_{\theta Y}^2\right)} + \frac{1}{\omega_{CX}^2 \left(1+4\zeta_{CX}{}^2\right)} \right]^{-1} \tag{2}$$

where:

$$\omega_X = \sqrt{K_X/m} \qquad \omega_{\theta Y} = \sqrt{K_{\theta Y}/mh^2} \tag{3}$$

are fictitious uncoupled natural frequencies of the system under rocking and swaying oscillation of the base, and the natural oscillation frequency of the undamped fixed-base structure is:

$$\omega_{CX} = \sqrt{k_X/m} \tag{4}$$

In Eqs. (1)-(2), ζ_X and $\zeta_{\theta Y}$ are the damping terms of the foundation in the vibration modes along *x*-axis and around *y*-axis, respectively, while ζ_{CX} is the damping ratio of the horizontal motion of the structure along *x*-axis. By approximating to unity the terms expressed as $(1+4\,\zeta^2)$, with ζ being any foundation or structural damping term, it is possible to rearrange Eqs. (1-2) into:

$$\begin{gathered} \tilde{\omega}_X{}^2 = \left[\frac{1}{\omega_X^2} + \frac{1}{\omega_{\theta Y}^2} + \frac{1}{\omega_{CX}^2} \right]^{-1} \Rightarrow \tilde{k}_X = \left[\frac{1}{K_X} + \frac{h^2}{K_{\theta Y}} + \frac{1}{k_X} \right]^{-1} \\ \tilde{\zeta}_X = \left(\frac{\tilde{k}_X}{K_X} \right) \zeta_X + \left(\frac{\tilde{k}_X h^2}{K_{\theta Y}} \right) \zeta_{\theta Y} + \left(\frac{\tilde{k}_X}{k_X} \right) \zeta_{CX} = \alpha_X \zeta_X + \alpha_{\vartheta Y} \zeta_{\theta Y} + \alpha_{CX} \zeta_{CX} \end{gathered} \tag{5a,b}$$

where $\tilde{k}_X$ is the translational stiffness along *x*-axis of the overall system. The same procedure may be applied to the motion along *y* direction.

Rewriting the expressions by Maravas et al. (2014) as in Eqs. (5a,b) has the advantage of offering an insight in the physics of the interaction phenomenon. The apparent damping is a linear combination of the damping ratios pertaining to the fixed-base structure, the swaying oscillation and the rocking oscillation of the foundation, weighted for the three coefficients α_X and $\alpha_{\theta Y}$, and α_{CX}. It can be verified that the sum of above coefficients is 1. As a result, if damping ratios ζ_X and $\zeta_{\theta Y}$ are equal to ζ_{CX}, the equivalent damping must be equal to ζ_{CX}. At the same time, if the soil-foundation system is very stiff compared to the structure, α_x and $\alpha_{\theta Y}$ are negligible, and the apparent damping of the replacement oscillator coincides again with ζ_{CX}.

Since stiffness and damping terms pertaining to the foundation are frequency-dependent, an iterative procedure is necessary to obtain the apparent damping of overall system. To this aim, frequencies and damping ratios corresponding to the natural circular frequency of the structure

ω_{ci} can be used as starting values, thereby calculating by means of Eq. (5a) a new value for the frequency $\tilde{\omega}_i^2$. Thereafter, new estimation of impedances may be obtained for $\tilde{\omega}_i^2$, until convergence. Generally, two or three iterations are sufficient to get accurate results.

The frequency-dependent terms ζ_i and $\zeta_{\theta j}$, associated to swaying and rotational modes of vibration of the pile foundation, can be easily obtained from the impedance functions:

$$\zeta_i = \frac{C_i(\omega)}{2K_i(\omega)} \qquad \zeta_{\theta j} = \frac{C_{\theta j}(\omega)}{2K_{\theta j}(\omega)} \tag{7}$$

where $C_i(\omega)$ and $K_i(\omega)$ are the imaginary and real parts of the dynamic stiffness associated to swaying along *i*-axis, while $C_{\theta j}$ and $K_{\theta j}$ are those associated to rocking around *j*-axis.

The rotational and horizontal stiffness components of the dynamic compliance have been evaluated again by means of code Dynapile 2.0 (1999). The analyses have been performed by referring to a symmetric layout characterized by a total of 76 piles (Figure 2b), all having a diameter of 2 m. Such idealization is a reasonable approximation for engineering purposes. While the modulus and damping of the soil are strain-dependent, studies (Kausel et al. 1976, 1978) have shown that most of non-linearity occurs as a result of the earthquake motion, and not as a result of soil structure interaction. Thus, the soil properties consistent with the levels of strain mobilized in the free-field soil response may be also used without further modification to account for the additional soil motion imposed by the oscillation of the structure. Consistently, the Dynapile analyses have been performed by assuming the same profile of the mobilized soil stiffness as for the kinematic interaction analysis (Fig. 1b).

The swaying and rocking damping ratio functions, ζ_i and $\zeta_{\theta j}$, are plotted in Figure 6. It is worthy of note that inverting *i* and *j* does not imply any significant variation of the damping ratio functions. The rocking damping ratio $\zeta_{\theta j}$ is practically independent of frequency, at least in the frequency range taken into account (0 to 1 Hz), i.e. where the predominant frequencies of the equivalent oscillator fall. On the other hand, the swaying damping ratios, ζ_i, significantly increase right beyond the fundamental translational frequency of the fixed-base structure.

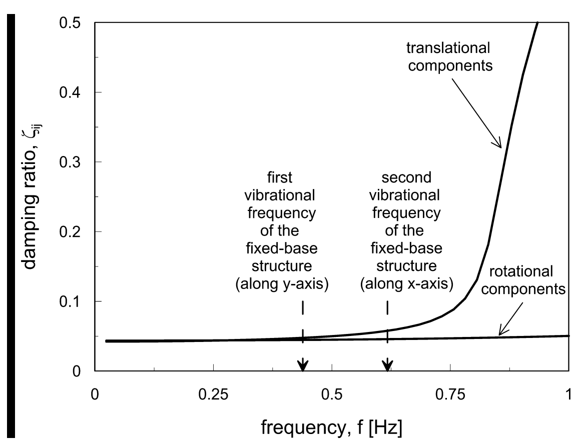

Figure 6. Frequency-dependent damping ratios

For the case at hand, the method by Maravas et al. (2014) provides the results shown in Table 1.

Table 1. Results of calculations according to Maravas et al. (2014)

$\tilde{T}_X$ (s)	$\tilde{f}_X$ (Hz)	$K_X(\tilde{f}_X)$ (MN/m)	$C_X(\tilde{f}_X)$ (MN/m)	$\zeta_X(\tilde{f}_X)$	$K_{\theta Y}(\tilde{f}_X)$ (MNm)	$C_{\theta Y}(\tilde{f}_X)$ (MNm)	$\zeta_{\theta Y}(\tilde{f}_X)$
1.88	0.531	$5.316 \cdot 10^3$	$5.455 \cdot 10^2$	5.13%	$1.312 \cdot 10^7$	$1.185 \cdot 10^6$	4.52%
$\tilde{T}_Y$ (s)	$\tilde{f}_Y$ (Hz)	$K_Y(\tilde{f}_Y)$ (MN/m)	$C_Y(\tilde{f}_Y)$ (MN/m)	$\zeta_Y(\tilde{f}_Y)$	$K_{\theta X}(\tilde{f}_Y)$ (MNm)	$C_{\theta X}(\tilde{f}_Y)$ (MNm)	$\zeta_{\theta X}(\tilde{f}_Y)$
2.47	0.405	$5.737 \cdot 10^3$	$5.343 \cdot 10^2$	4.66%	$1.373 \cdot 10^7$	$1.221 \cdot 10^6$	4.45%

The swaying and rocking damping ratios are very close to the value (5%) of the fixed-base structure. In addition, the terms (ζ_i, $\zeta_{\theta j}$) are negligible with respect to ζ_{Ci}, because the stiffness of the soil-foundation system is very high compared to that of the structure. As a result, the apparent damping did not change significantly for both x- and y-axis.

Results

The reduction of the seismic actions achieved by taking into account the free-field site response and the dynamic soil-structure interaction in the problem at hand is presented in Figure 7. The average spectrum, computed from the accelerograms resulting from the seismic response analyses by assuming $\tilde{\zeta}_X = \tilde{\zeta}_Y = 5\%$, is compared to the spectrum for ground type D.

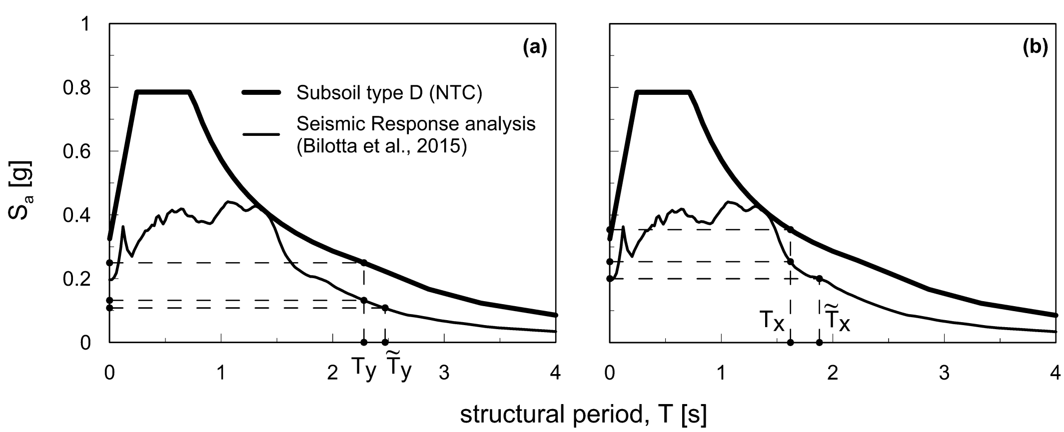

Figure 7. Reduction of the inertial actions: first (a) and second (b) building vibration modes

As an effect of the seismic response, at the first vibration mode (T_y = 2.28 s) the spectral acceleration on the fixed-base structure is reduced of 47.2 % with respect to that specified by the code. An additional reduction (9.5%) of the inertial action is due to the increase in the structural period induced by the inertial interaction ($\tilde{T}_Y$ = 2.47s). For the second vibration mode (T_x = 1.62 s), the reduction due to seismic site response is equal to 28.3% with a further significant reduction of 15.1% associated to the foundation compliance.

Summarizing, the overall reduction of inertial action due to both seismic response and soil-structure interaction is 56.8% along *y*-axis and 43.5% along *x*-axis.

Conclusions

The seismic actions on a tall public building in Napoli (Italy) were evaluated considering seismic site effects and dynamic soil-structure interaction. The following results were obtained:

- pile filtering does not affect the building response, due to its large fundamental period;
- the swaying and rocking components of the foundation impedance permitted to evaluate the increase of the structural period of the compliant base system, that is 1.08 and 1.16 for the first and second modes, respectively;
- due to free-field soil response and inertial soil-structure interaction, the spectral accelerations are overall reduced by 57% and 43% for the first and second vibration modes.

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